BEHAVIOUR OF TALL BUILDINGS DURING THE CARACAS EARTHQUAKE OF 1967

by

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Synopsis

The behaviour of buildings and other structures during the 1967 Caracas Earthquake is analysed. Static and dynamic characteristics, such as coefficients of resistance and natural frequencies, are computed and compared with the observed damages. In several cases natural frequencies were also measured on the site.

The intensity of the earthquake in different areas is estimated on the basis of the maximum displacements suffered by the structures, the forces necessary to produce rupture and the amount of damage observed. The influence of soil conditions is briefly discussed.

The results obtained may contribute in assessing the adequacy of the design rules concerning reinforced concrete tall buildings and, particularly, of the seismic factors usually adopted in the codes.

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1 - INTRODUCTION

The study of the structural behaviour of buildings under strong earthquakes yields results of paramount importance for judging design rules. This is particularly true in the case of the Caracas earthquake of 29th July 1967, which struck a big town with more than 1000 buildings over 10 stories. Due to the earthquake several buildings (5 of which more than 10 stories high) collapsed and many others were considerably damaged.

In the survey prepared by the Ministerio de Obras Publicas[1], 2300 cases of damage were recorded in the Caracas area, from which 27% were considered important. One half of these cases concern one or two story houses and the other half higher buildings. Sixty-eight structures, 17 of which were buildings, collapsed or were damaged beyond repair.

In the present report 51 cases are referred. A quantitative study of the structural behaviour is presented for 22 structures, 14 of which are buildings (9 having 10 stories or more). Four of the collapsed buildings are studied.

Each structure is analysed on the basis of information obtained in the direct inspection of the site, on results of tests of materials performed at the IMME[2] (Instituto de Materiales y Modelos Estructurales) of the Universidad Central de Venezuela and on the available plans. It was possible in each case to compute the values of the horizontal forces which if applied at a given level and in a given direction would produce collapse. By dividing these forces by the weight of the structure above the considered level, coefficients of resistance were obtained. These resistance coefficients can be directly compared with the seismic coefficients prescribed in the codes if they are computed according to the same design rules.

Also in each case the natural frequencies of each building are computed and compared with experimental values when available.

The intensity of the earthquake at the different locations (expressed in power spectral density of acceleration) is estimated on the basis of the maximum displacements undergone by the structures, on the basis of the ultimate forces they can resist and by judging the observed damages. The increase in seismic intensity due to the influence of soil conditions is also briefly studied.

By comparing the observed damages with the results of the computations, conclusions are drawn concerning the intensity of seismic forces and the adequacy of design rules.

No general description of the earthquake and of its consequences is attempted in the present paper. Particular attention is called to recent publications on the matter (3 to 10)

2 - BASIC ASSUMPTIONS

The interpretation of the results concerning structural behaviour is based principally on the comparison between the amount of observed damage and the coefficients of resistance to horizontal forces. The natural frequencies corresponding to the type of structural behaviour assumed for computing the coefficients of resistance are also taken into consideration.

Damage is classified into 4 groups: collapse, important damage, small damage, no damage. This classification tries not only to express the amount of damage observed but also to express a subjective probability of collapse in correspondence with the observed damage. The structures with no damage and small damage are considered to have a small probability of collapse. Those structures assumed to have a probability of collapse near to 1 are classified in the important damage group. Thus the structures with important damages present a clear mechanism of collapse and local damage or residual displacements showing that their ductility is practically exhausted.

For determining the static horizontal forces necessary to produce the collapse of the reinforced concrete structures, and thus the resistance coefficients, the following hypotheses were adopted concerning the type of structural behaviour and the ultimate strength of reinforced concrete elements.

2.1 - Structural behaviour

The observation of the damaged buildings suggested the 6 main types of structural behaviour indicated in fig.1. Thus the ultimate forces necessary to produce collapse were computed according to these idealizations.

Structural behaviour type 1 corresponds to the formation of plastic hinges at the top and at the bottom of the columns at one of the stories. According to ultimate design principles the horizontal force which produces collapse is obtained by adding the ultimate forces of the different columns. This type of structural behaviour occurs when the beams of the floors are considerably more rigid and resistant than the columns or when the walls at the stories near the considered one can prevent the rotation of the joint blocks. This type of behaviour often occurred in buildings with a small amount of partitions at one level (in general the ground floor).

Structural behaviour type 2 corresponds to a collapse mechanism affecting two stories, with formation of plastic hinges in the columns and in the floor structure (beams or slabs). This type of behaviour occurs when the structure of one floor cannot resist the moments transmited by the columns. This is a very frequent case in Caracas, where structures with transverse beams only are very often used. Generalizing this type of behaviour it would be possible to consider the formation of plastic hinges in more than two stories but such schemes are no more realistic. In fact plastic hinges can only form at several stories under too large horizontal displacements.

Structural behaviour type (3) corresponds to the elastic behaviour of framed structures. When the buildings are formed by parallel frames of unequal stiffness, accurate results are only obtained if compatibility is imposed to the horizontal displacements of the different frames at the different levels. This type of frame behaviour only occurs when the structural contribution of partitions is small.

Structural behaviour type (4) corresponds to the association of shear walls with a framed structure. This type of behaviour is studied using the usual frame analysis programs but introducing segments of infinite rigidity to reproduce the dimensions of the joint blocks.

Structural behaviour of types (5) and (6) correspond to cantilevers with distributed mass and concentrated mass at the top, respectively.

It must be noted that the type of structural behaviour may change as a function of the intensity of the seismic loading.

2.2 - Ultimate strength of reinforced concrete elements

The ultimate strength of reinforced concrete elements under pure bending or combined axial compression and bending is computed according to the Recommendations of European Committee for Concrete (11). These Recommendations were adopted as a basis for the Portuguese Code for Reinforced Concrete Structures (12) and thus the tables prepared for this code were used.

The basic assumptions consist in considering for concrete a stress-strain diagram defined by a parabola with the vertex at point (σ_b^* , 2°/oo) with $\sigma_b^* = \frac{\sigma_{kb}}{1.5}$, σ_{kb} being the characteristic (5% fractile) of the 28 days cylinder strength of concrete. This cylinder strength is taken as 85% of the cube strength.

For mild steel the elasto-plastic stress-strain diagram is defined by a yielding stress $\sigma_a^* = \frac{\sigma_{ka}}{1.15}$, σ_{ka} being the characteristic yielding stress of the steel. For work-hardened steel the diagram is curved and defined by the characteristic limit of proportionality at 0.2%, divided by the minoration factor of 1.15. The ultimate strain for steel is 10°/oo.

For simplifying the analysis of the data the concrete qualities are classified in 4 categories: B180, B225, B250, B300, the reference numbers indicating the characteristic cube strength. The steel qualities are designated by A24 and A40 corresponding to the characteristic yielding stress of 2 400 kg/cm^2 and the characteristic limit of proportionality at 0.2% of 4 000 kg/cm^2.

The interaction diagrams of fig.2 indicate the ultimate values of axial force, N , and bending moment M , for symmetrically reinforced rectangular columns with total percentages of steel, ϖ_t = 0.5 , 1.0 and 2.0% for the following combinations of the mechanical properties of the materials: i) A24, B180 and ii) A40, B300 (curves (1)).

The interaction diagrams represented by curves (2) correspond to the ACI 318-63 Building Code (Chapter 19, Combined Axial Compression and Bending - Ultimate Strength Design). Considering elastic behaviour and working stress design, curves (3) are obtained. Curves (3) refers to the allowable stresses currently used in seismic design, which are 33% higher that those for permanent and live loads.

A comparison of curves (1) and (2) shows that the CEB or ACI criteria are not very different. ACI criteria are about 20 to 30% safer than the Portuguese Code of 1967. The figure emphasizes the inadequacy of working stress design for combined axial force and bending moment. In fact the ratio of the bending moments indicated by curves (1) and (3) vary between about 1 and 2.

2.3 - Load factors

For comparing the coefficients of resistance with the seismic coefficients indicated in the codes attention has to be paid to the values of the specified load factors. In fact several codes take, for earthquake loading, load factors equal to 1 but other codes prescribe load combinations where earthquake loads are multiplied by factors different from 1. This is, for instance the case of the 1967 revision of the American code "Recommended Lateral Forces Requirements" (13). This code indicates that load resisting frames must be designed for the following loads:

$$\text{i) } U = 1.40\,(D + L + E)$$
$$\text{ii) } U = 0.9\,D + 1.25\,E$$

where D — permanent load, L — live load, E — earthquake load.

The safety concepts in which these combinations are based are not clear. From a probabilistic point of view it does not seem justifiable to consider earthquake loads multiplied by 1.4 in expression i) and by 1.25 in expression ii). Anyhow, when comparing coefficients of resistance with seismic coefficients specified in this American code, the indicated values of the load factor have to be considered. Also if working stresses are used (even if increased by 33%) the ultimate bending moments obtained are in general much smaller than those determined in ultimate strength design. Thus the values of coefficients of resistance determined in the present paper can only be directly compared with the seismic coefficients of the codes that use a load factor equal to 1 and ultimate load design.

2.4 - Natural frequencies

The interpretation of the results is based on the natural frequencies of the structures, which were computed and compared with experimentally determined values.

For computing the natural frequencies structural schemes analogous to those used for determining the coefficients of resistance were adopted.

In the case of structures of types (1) and (2) deformability of one or two stories only was considered, the remaining being assumed to be rigid. The behaviour of the deformable stories is assumed to be perfectly elastic and thus linear one-degree-of-freedom oscillators are obtained. For structures of types (3) and (4) the natural frequencies are determined by the Rayleigh method and in some cases also by modal analysis. In these cases the effect of non structural walls is disregarded and masses are concentrated at floor levels. In cases (5) and (6) a given amount of foundation deformability is assumed in order to obtain values of the natural frequencies in agreement with the measured ones.

3 - STUDIED CASES

The studied cases are summarized in Tables I to III ,which refer to structures at Palos Grandes and Altamira in the town of Caracas (Table I), at Caraballeda (Table II) and at Naiguatá and Playa Grande (Table III), fig. 3. At Palos Grandes, Altamira and Caraballeda there are deep layers of alluvial soil. At Naiguatá and Playa Grande the thickness of soft soils is supposed to be much smaller.

The qualities of the materials were experimentally determined in some cases (2). When test results were not available the qualities indicated in construction plans were adopted. When no indication was given in the plans the qualities A24 and B180 were assumed.

The tables indicate the approximate orientation of the structures and the ratios H/L of height to longitudinal and transverse dimensions at the base.

The amount of damage and the type of structural behaviour are classified as indicated in 2). The estimated importance of the structural contribution of the partitions is also included.

The coefficients of resistance, and the computed and measured natural frequencies in each direction are finally indicated. Natural frequencies were measured with a Dr. Baule's B4h displacement meter produced by Höttinger (14).

Fig. 4 to 13 show some of the collapsed or damaged structures.

Fig. 14 compares the importance of damage, the coefficients of resistance and the computed natural frequencies at the Palos Grandes--Altamira area and at Caraballeda, Naiguatá and Playa Grande.

Table IV summarizes the available data on 29 cases that were only summarily observed and for which the coefficients of resistance could not be computed. The natural frequencies indicated for all these structures were measured in some cases before and in other cases after the structures had been repaired. The type of structural behaviour indicated corresponds to the observed damage. Thus, when no damage was observed no indication is given.

For the Residence Union and Mene Grande buildings (cases 41 and 47) which are H-shaped in plan, the torsion vibration mode was detected. This mode was particularly obvious in Residence Union.

Fig. 15 relates the natural periods measured in longitudinal and transverse directions to the number of stories.

4 - INTERPRETATION OF THE RESULTS

4.1 - Intensity of the earthquake

For the interpretation of the results it would be of paramount importance to estimate the intensity of the earthquake in different areas and to study the influence of soil conditions. As no strong motion instrumental records are available this is a difficult task.

A way to estimate the power spectral intensity of acceleration is based on the maximum displacements observed in some structures. According to previous results (15) the mean maximum displacement of a one-degree-of-freedom oscillator with 5% critical damping, expressed in cm, is approximately given by

$$\delta = \frac{6}{f}\sqrt{\frac{S}{700}} = 0.23\,\frac{\sqrt{S}}{f} \quad \ldots\ldots\ldots\ldots \quad 1)$$

where f (Hz)* is the natural frequency of the structure, S($cm^2 s^{-4}$/Hz) is the power spectral density of acceleration of the considered earthquake and 700 $cm^2 s^{-4}$/Hz is the intensity of the standard earthquake (30 s of N-S El-Centro 1940 record). Thus S can be estimated by

$$S = 19\ \delta^2\ f^2 \quad \ldots\ldots\ldots\ldots \quad 2)$$

The computations concerning the Macuto-Sheraton Hotel main building (Table II, ref. no. 14) indicate that for a horizontal transverse force equal to 0.22 of the weigth of the structure (force necessary to produce yielding of the columns) the elastic displacement above the 4th story is of about 4.5 cm. Tests on reinforced concrete models to scale 1/5.5 of the collapsed columns have shown that the amount of damage observed in the columns corresponds to residual displacements of about 3.5 cm Thus the maximum displacement due to the earthquake can be estimated as 8 cm. By using expression 2) and by considering the natural frequency of 1.4 Hz. a spectral density of 2 400 $cm^2 s^{-4}$/Hz is obtained.

For the Macuto-Sheraton Pergolas (table II, ref. no. 22 and 23) the indicated expression is no longer valid as the effect of gravity can no more be disregarded, as indicated by Husid (16). By performing a dynamic non-linear analysis taking in consideration the effect of gravity, a maximum displacement of 24 cm is reached (for t=13 s) when Pergola 1 is subjected to the accelerogram of N-S El-Centro 1940 earthquake having its ordinates multiplied by $\sqrt{1.5}$.

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* – Hz - Herz - cycles per second

Another way to estimate the power spectral density of acceleration consists in computing the response of systems with a number of degrees of freedom sufficient to secure an accurate representation of the structural behaviour and to verify at what values of the spectral density of acceleration the observed amount of damage or collapse is attained. Computations of this type considering non-linear behaviour were performed for Mansion Charaima (table II, ref. no. 17) and show that for a power spectral density of 700 cm^2s^{-4}/Hz rupture of the columns is attained at the 7th story due to a whiplash effect .

The dynamic analysis of Petunia II building (table I, ref. no. 8) performed considering the linear behaviour of a 11 degree of freedom system on an elastic foundation indicates that collapse by compression failure of the external columns occurs for a power spectral density of acceleration of about 500 cm^2s^{-4}/Hz.

The seismoscope situated at Observatorio Cajigal ($T = 0,7$ s , $\eta = 0,029$) founded on rock, suffered a maximum acceleration of 200 cm^2/s (18). This acceleration corresponds to a constant spectral density of about 150 cm^2s^{-4}/Hz over the frequency range 0-5 Hz.

Finally comparisons of the coefficients of resistance with the observed damages, and the observed damages themselves give qualitative indications on the intensity of the earthquake.

It can be concluded that the intensity of the earthquake was much amplified in the Caraballeda area where an intensity considerably higher than the standard one was attained. This amplification is also apparent at Palos Grandes - Altamira but the intensity in this area is assumed to be lower than 700 $cm^2\ s^{-4}/Hz$. In Caracas outside this area the intensity of the earthquake was even smaller.

It must be noticed that, according to the magnitude of the earthquake (about 6.3 Richter) and the distance to the epicenter(70 km) (18), an intensity smaller then the one of El-Centro would be expected in Caracas.

4.2 - Soil conditions

Available data on soil conditions in the studied areas indicate the occurrence of consolidated alluvial layers at Palos Grandes-Altamira with a thickness over 80 m and of softer alluvial layers reaching to at least 70 m in Caraballeda area. These thicknesses obtained through recent drilling (19) exceed the ones mentioned in previous geological information (7).

Spectral analysis of microtremors registered by Grases (20) at three different places of Palos Grandes-Altamira show a predominant frequency of about 0.7 Hz. This value, that is near to the natural frequencies of the tall buildings existing in this area, suggests a local selective ampplifying effect(21). The existence of softer layers at Caraballeda may be responsable for the more severe damage that occured in this area. However this important problem deserves further study.

4.3 - Behaviour of buildings

Fig. 14 shows that at Palos Grandes and Altamira area the buildings that collapsed or suffered important damages had coefficients of resistance below about 0.08.

The fact that Petunia II building (table I, ref. no. 8), having a coefficient of resistance of 0.10, underwent important damage can in part be explained by its cantilever behaviour which corresponds to a lower ductility than the one corresponding to frame behaviour.

In the Caraballeda area the buildings that collapsed or suffered important damages had coefficients of resistance below about 0.22. In Naiguatá and Playa Grande this limit decreases to about 0.12.

The influence of the natural frequency on the amount of damage is not directly apparent in fig. 14. This may be due to the relatively small range of natural frequencies in the structures and also to the fact that the influence of this factor is not as important as indicated by the proportionality between seismic coefficient and natural frequency.

Fig. 15 relates the measured natural periods, T, in longitudinal and transverse directions to the number of stories, N. The mean value of T/N is 0.07 for the 64 cases considered, the coefficient of variation, V, being 35%.

Correlations of the type $T(s) = K \frac{H}{\sqrt{B}}$, where K is a constant, H the height and B the depth (m) of the buildings were also considered the following results being obtained.

H/B	Number of cases	K	V(%)
> 2	29	0,082	14
< 2	19	0,123	22
all	48	0,098	29

Particular reference is due to the very good behaviour of Plaza I building (table IV, ref.no.42) having a reinforced concrete shear walls structure.

A description of some accidents due to faulty details is not included in the present paper but can be found in several of the indicated references.

5 - CONCLUSIONS

The main conclusions derived are the following.

5.1 - The intensity of the earthquake was very different in the various areas of Caracas and nearby locations. The following approximate estimates of the power spectral density of acceleration expressed in $cm^2 s^{-4}/Hz$, can be indicated:

Caraballeda	≃ 1 500
Playa Grande-Naiguatá	≃ 700
Palos Grandes-Altamira	≃ 500
Cajigal	≃ 150

5.2 - The variation of seismic intensity seems mainly due to different soil conditions. Even so present information does not permit a quantitative assessment of the relationship between the observed intensity and the soil conditions.

5.3 - The comparison between the computed coefficients of resistance and the observed damages shows that the buildings that collapsed or suffered important damage had coefficients of resistance below about 0.22 in Caraballeda, 0.12 in Naiguatá and Playa Grande, and 0.08 in Palos Grandes-Altamira.

5.4 - In the observed cases, the influence of the natural frequency of vibration on the relation between coefficients of resistance and amount of damage does not appear to be important.

5.5 - Due to the fact that most structures were formed by frames, the influence of the type of structure and of the correspondent ductility on the relation between coefficients of resistance and amount of damage was not clearly detected. It would be of great value to obtain further data on this matter.

5.6 - The collapses can in general be explained by the small values of the coefficients of resistance and are not attributed to poor construction or faulty detailing. The coefficient of resistance are particularly low for those structures that in one direction had no beams but only slender flat slabs.

5.7 - The presented results may contribute in assessing the adequacy of seismic factors usually adopted. Attention must be paid to the fact that the indicated coefficients of resistance can only be directly compared with the seismic factors indicated by those codes that consider ultimate design and load factors equal to 1.

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REF. NUMBER	DESIGNATION Nº. OF STORIES HEIGHT, H (m)	QUALITIES OF MATERIALS	DIRECTIONS	H/L	AMOUNT OF DAMAGE	TYPE OF STRUCTURAL BEHAVIOUR	CONTRIBUTION OF PARTITIONS	COEFFICIENT OF RESISTANCE	NATURAL FREQ.(c/s)	
									COMP.	MEAS.
TABLE I - STUDIED STRUCTURES AT PALOS GRANDES AND ALTAMIRA										
1	PALACE CORVIN 10 st., H=32 (2 PARALLEL BUILDINGS)	A 24 *	LONG. N-S	1.3	COLLAPSE OF BUILDING 1	2	NONE	0.05	0.8	–
2		B 180 *			IMPORTANT IN BUILDING 2	2	SMALL	0.05	0.8	1.0
3			TRANSV. E-W	4.1	SMALL IN BUILDING 2	1	IMPORTANT	0.10	2.6	1.4
4	MIJAGUAL 11 st., H=32	A 40 *	LONG. E-W	1.5	COLLAPSE	2	NONE	0.06	1.4	–
–		B 225 *	TRANSV. N-S	3.7		2	NONE	0.08	1.6	–
–	NEVERI 12 st., H=35	A 40 *	LONG. E-W	1.7	COLLAPSE	2	NONE	0.09	1.7	–
5		B 225 *	TRANSV. N-S	3.5		2	NONE	0.06	0.9	–
6	PETUNIA I 15 st., H=41	BEAMS A 40 **	LONG. E-W	1.4	SMALL	1	NONE	0.14	2.6	1.0
7		COLUMNS A 24 ** B 250 **	TRANSV. N-S	5.5	IMPORTANT	3	NONE	0.05	0.8	0.7
8	PETUNIA II 21 st., H=57	A 40 **	LONG. N-S	4.7	IMPORTANT	5	IMPORTANT	0.10	0.6	0.7 (1)
–		B 225 **	TRANSV. E-W	4.9		–	IMPORTANT	–	–	0.8 (1)
9	SAN BOSCO 14 st., H=40	A 24 **	LONG. N-S	1.3	IMPORTANT	2	NONE	0.04	0.8	–
–		B 225 **	TRANSV. E-W	2.9		2	NONE	0.06	1.2	–
–	MARCO AURÉLIO 9 st., H=25	A 24 ***	LONG. E-W	1.0	IMPORTANT	–	NONE	–	–	–
10		B 180 ***	TRANSV. N-S	3.2		2	NONE	0.08	1.2	–
–	ATLANTIC, CENTRAL BLOCK 9 st., H=29	A 24 ***	LONG. N-S	2.0	–	–	–	–	–	–
11		B 180 **	TRANSV. E-W	4.9	SMALL	4	NONE	0.08 (2)	1.5	–
12	HONGO CVP H = 4.3	A 24 ** B 225 **	–	–	NO DAMAGE	6	–	0.14	0.8	0.7
TABLE II - STUDIED STRUCTURES AT CARABALLEDA										
REF. NUMBER	DESIGNATION Nº. OF STORIES HEIGHT, H (m)	QUALITIES OF MATERIALS	DIRECTIONS	H/L	AMOUNT OF DAMAGE	TYPE OF STRUCTURAL BEHAVIOUR	CONTRIBUTION OF PARTITIONS	COEFFICIENT OF RESISTANCE	NATURAL FREQ.(c/s) COMP.	MEAS.
13	MACUTO-SHERATON HOTEL, MAIN BUILDING 10 st., H = 38	A 24 *	LONG.N	0.8	SMALL ABOVE THE 4th STORY	1	IMPORTANT	0.23	1.2	–
14		B 300 *	TRANSV.	2.1	IMPORTANT BELOW 4th STORY	4	NONE	0.22	1.4	–
15	MANSION CHARAIMA 10 st., H = 31	A 24 *	LONG.	0.6	SMALL (1st STORY)	2	NONE	0.13	0.6	1.3 (3)
16		B 180 *	TRANSV.	2.1	SMALL (1st STORY)	2	NONE	0.13	0.6	1.1 (3)
17					COLLAPSE (8th STORY)	1	NONE	0.13	1.0 (3)	–
18	BLOQUE 32 URB. CARIBE 3 st., H = 10	A 24 ***	LONG. EW	0.4	COLLAPSE (1st STORY)	1	NONE	0.20	2.2	–
19		B 180 ***	TRANSV. N-S	0.8		1	NONE	0.20	2.2	–
20, 21	BLOQUES 38 AND 21 D, URB. CARIBE st., H=10	A 24 ***	LONG. N-S	0.4	COLLAPSE (2 BUILDINGS)	1	IMPORTANT	0.12	2.4	–
		B 180 ***	TRANSV. E-W	1.4		1	SMALL	0.10	2.1	–
22	MACUTO SHERATON HOTEL, PERGOLAS 1 AND 2, H=2.7	STEEL* $\sigma_{0.2}$ = 3400kg/cm^2	–	–	IMPORTANT, 30cm N-S RESIDUAL DISPLACEMENT	1	–	0.20	0.9	–
23			–	–	IMPORTANT, 27cm N-S RESIDUAL DISPLACEMENT	1	–	0.16	1.2	–
24	SHELL SERVICE STATION - SINGLE ROOF H = 4	A 24 ** B 225 **	– –	– –	COLLAPSE	6	–	0.20	1.0	–
TABLE III - STUDIED STRUCTURES AT NAIGUATA AND PLAYA GRANDE										
25	BALNEARIO NAIGUATA, BUILDING 1 st., H = 8	A 24 **	LONG. N-S	0.2	SMALL	1	NONE	0.14	1.0	–
26		B 180 **	TRANSV. E-W	0.3	IMPORTANT, 22cm E-W RESIDUAL DISPLACEMENT	1	NONE	0.12	1.0	–
27	BALNEARIO NAIGUATA, BUILDING 2 st., H = 8	A 24 **	LONG. N-S	0.2	NO DAMAGE	1	SMALL	0.11 (4)	3.2	–
		B 180 **	TRANSV. E-W	1.6		1	SMALL	0.11	3.2	–
28	BALNEARIO NAIGUATA, TOWER H = 30	A 24 ** B 180 **	–	9.4	NO DAMAGE	5	NONE	0.60	0.8	–
29	CLUB PLAYA GRANDE, SINGLE ROOF 1 st., H = 4	A 24 **	LONG. E-W	–	NO DAMAGE	1	–	0.16	1.1	–
30		B 225 **	TRANSV. N-S	–	SMALL		–	0.20	1.5	–

* EXPERIMENTALLY DETERMINED; ** ACCORDING TO THE CONSTRUCTION PLANS; *** ASSUMED.

(1) DETERMINED WITH REINFORCING SCAFFOLDING INSTALLED.

(2) CORRESPONDS TO THE HORIZONTAL FORCE NECESSARY TO PRODUCE FAILURE IN THE DAMAGED COLUMNS. THERE IS AN IMPORTANT RESERVE OF RESISTANCE DUE TO THE SHEAR-WALLS AND COLUMNS NEARBY.

(3) CORRESPONDS TO THE 7th STORY BUILDING REMAINING AFTER THE COLLAPSE OF THE UPPER STORIES.

(4) REFERS TO THE 2nd STORY.

TABLE IV - DATA ON OBSERVED STRUCTURES

REF. NUMBER	DESIGNATION Nº. OF STORIES HEIGHT, H (m)	LOCATION	DIRECTIONS	H/L	MEASURED NAT. FREQUENCY (c/s)	TYPE OF STRUCTURAL BEHAVIOUR	OBSERVED DAMAGE
31	LASSIE 9 st., H = 26	PALOS GRANDES	LONG. E-W	0.6	2.17	-	NO DAMAGE.
			TRANSV.N-S	2.6	1.76	-	NO DAMAGE.
32	ICABARU 16 st., H = 50	PALOS GRANDES	LONG. E-W	1.4	1.61 [1]	3	NO STRUCTURAL DAMAGE. IMPORTANT DAMAGE IN PARTITIONS.
			TRANSV.N-S	5.0	1.00 [1]	3	
33	CASTILLETE 18 st., H = 50	PALOS GRANDES	LONG. N-S	2.0	0.83	3	NO STRUCTURAL DAMAGE. IMPORTANT DAMAGE IN PARTITIONS UP TO 9th STORY
			TRANSV.E-W	7.1	0.67	-	NO DAMAGE.
34	LE ROC EAST BLOCK 14 st., H = 42	PALOS GRANDES	LONG. N-S	2.5	0.95	3	NO STRUCTURAL DAMAGE. IMPORTANT DAMAGE IN PARTITIONS UNTIL THE 5th STORY.
			TRANSV.E-W	2.5	0.94	-	
35	LE ROC WEST BLOCK 14 st., H = 42	PALOS GRANDES	LONG. E-W	2.4	0.95	2	FAILURES ON BEAMS AT THE 1st AND 2nd STORIES. IMPORTANT DAMAGE IN PARTITIONS.
			TRANSV.N-S	3.5	0.67	2	
36	RESIDENCIAS GUIPELIA-SUR 19 st., H = 53	PALOS GRANDES	LONG. E-W	3.6	0.83 [2]	2	SOME STRUCTURAL DAMAGE.
			TRANSV.N-S	5.6	0.81 [2]	2	
37	RESIDENCIAS GUIPEL.-NORTE 18 st., H = 50	PALOS GRANDES	LONG. E-W	3.3	0.95 [2]	2	SOME STRUCTURAL DAMAGE. DAMAGE IN PARTITIONS TILL THE 5th STORY.
			TRANSV.N-S	5.3	0.63 [2]	2	
38	IPASOLAZ 12 st., H = 31	PALOS GRANDES	LONG. N-S	1.2	0.83	3	SOME STRUCTURAL DAMAGE. DAMAGE IN PARTITIONS TILL THE 5th STORY.
			TRANSV.E-W	1.6	1.08	3	
39	PASAQUIRE 12 st., H = 34	PALOS GRANDES	LONG. E-W	1.3	1.30	3	NO STRUCTURAL DAMAGE. DAMAGE IN PARTITIONS TILL THE 6th STORY.
			TRANSV.N-S	1.4	0.92	3	
40	BLUE PALACE 18 st., H = 51	PALOS GRANDES	LONG. N-S	2.7	0.91 [1]	3	SOME STRUCTURAL DAMAGE. DAMAGE IN PARTITIONS TILL THE 12th STORY.
			TRANSV.E-W	3.4	0.91 [1]	-	
41	RESIDENCIAS UNION 16 st., H = 48	PALOS GRANDES	LONG. E-W	1.1	2.50 [3]	3	IMPORTANT STRUCTURAL DAMAGE. DAMAGE IN PARTITIONS TILL THE 7th STORY.
			TRANSV.N-S	1.4	2.50 [3]	3	
42	PLAZA UNO 19 st., H = 54	PALOS GRANDES	LONG. N-S	1.6	1.37	5	NO DAMAGE.
			TRANSV.E-W	3.1	1.72	5	
43	VISTA HERMOSA 10 st., H = 32	PALOS GRANDES	LONG. E-W	0.9	2.04	-	NO DAMAGE.
			TRANSV.N-S	2.3	1.62	-	
44	RESIDENCIAS HAWAII 6 st., H = 17	PALOS GRANDES	LONG. E-W	0.8	2.27	-	NO DAMAGE.
			TRANSV.N-S	2.6	2.17	-	
45	COVENT GARDEN 14 st., H = 40	PALOS GRANDES	LONG. E-W	1.0	1.18 [2]	3	SOME STRUCTURAL DAMAGE. DAMAGE IN PARTITIONS TILL THE 5th STORY.
			TRANSV.N-S	2.9	0.85 [2]	-	
46	HOUSTON 7 st., H = 20	PALOS GRANDES	LONG. E-W	0.6	3.04	-	NO DAMAGE.
			TRANSV.N-S	2.2	2.22	-	
47	MENE GRANDE 16 st., H = 58	PALOS GRANDES	LONG. E-W	1.4	0.83 [1]	3	STRUCTURAL DAMAGE IN THE COLUMNS IN THE 1st STORY. GENERAL DAMAGE OF PARTIT.
			TRANSV.N-S	1.5	1.11 [1]	-	
48	ROYAL 11 st., H = 31	PALOS GRANDES	LONG. E-W	1.1	1.16	2	SOME STRUCTURAL DAMAGE. DAMAGE IN PARTITIONS TILL THE 6th STORY.
			TRANSV.N-S	2.6	0.86	2	
49	HOTEL MONTSERRAT 7 st., H = 20	ALTAMIRA	LONG. N-S	1.1	1.72	3	NO DAMAGE.
			TRANSV.E-W	1.2	2.23	3	
50	MI VEGUITA 17 st., H = 50	SEBUCAN	LONG. N-S	2.4	1.25 [1]	3	SOME STRUCTURAL DAMAGE. DAMAGE IN PARTITIONS TILL THE 4th STORY.
			TRANSV.E-W	3.5	1.00 [1]	3	
51	LAS AMERICAS 17 st., H = 50	SEBUCAN	LONG. E-W	1.9	0.91	3	NO DAMAGE.
			TRANSV.N-S	5.0	1.00	3	
52	ZULIA 17 st., H = 54	BOLEITA	LONG. E-W	2.1	0.94	3	NO DAMAGE.
			TRANSV.N-S	5.4	1.03	3	
53	PHELPS 27 st., H = 84	LOS CAOBOS	-	3.4	0.66	4	NO DAMAGE.
			-		0.65	4	
54	POLAR 20 st., H = 60	LOS CAOBOS	LONG. E-W	2.7	0.87	4	NO DAMAGE.
			TRANSV.N-S	7.0	0.62	4	
55	MALVAROSA 7 st., H = 20	EL BOSQUE	LONG. E-W	0.7	2.50	3	NO DAMAGE.
			TRANSV.N-S	2.2	2.26	3	
56	BAHIA DEL MAR 13 st., H = 40	CARABALLEDA	LONG. E-W	0.7	0.60	3	STRUCTURAL DAMAGE ON THE REINFORCED CONCRETE ELEVATOR CORE. IMPORTANT DAMAGE IN PARTITIONS TILL THE 7th STORY.
			TRANSV.N-S	5.3	0.58	2	
57	LAGUNA BEACH 14 st., H = 40	CARABALLEDA	LONG. E-W	0.7	0.50	3	SOME STRUCTURAL DAMAGE. DAMAGE IN PARTITIONS TILL THE 10th STORY.
			TRANSV.N-S	3.4	0.71	3	
58	EL MIRADOR 24 st., H = 66	NAIGUATA	LONG. E-W	1.8	0.74 [4]	3	NO DAMAGE.
			TRANSV.N-S	5.8	0.59 [4]	4	
59	SANTA MARIA 16 st., H = 47	NAIOUATA	LONG. E-W	2.4	0.97	3	NO STRUCTURAL DAMAGE - SMALL DAMAGE IN PARTITIONS.
			TRANSV.N-S	3.4	1.10	3	

(1) AFTER REPAIRS.
(2) REINFORCING SCAFFOLDING INSTALLED.
(3) A TORSION MODE WAS DETECTED WITH 0.5 c/s NATURAL FREQUENCY.
(4) BUILDING HAVING STRUCTURE COMPLETED AND PARTITIONS UNDER CONSTRUCTION.

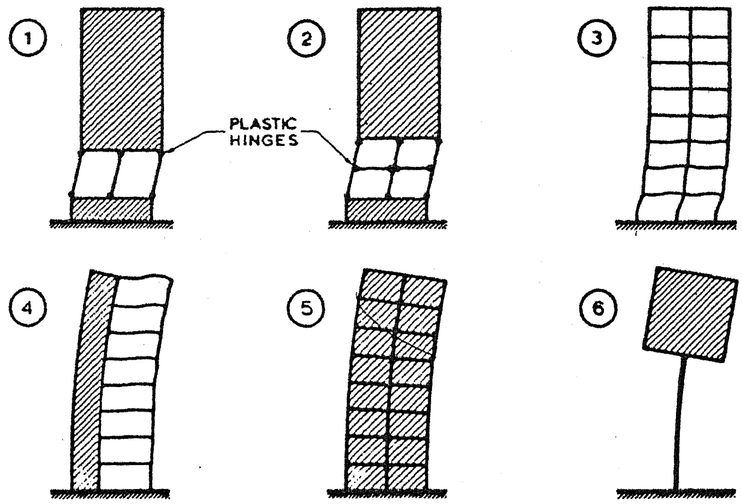

Fig. 1 - Types of structural behaviour

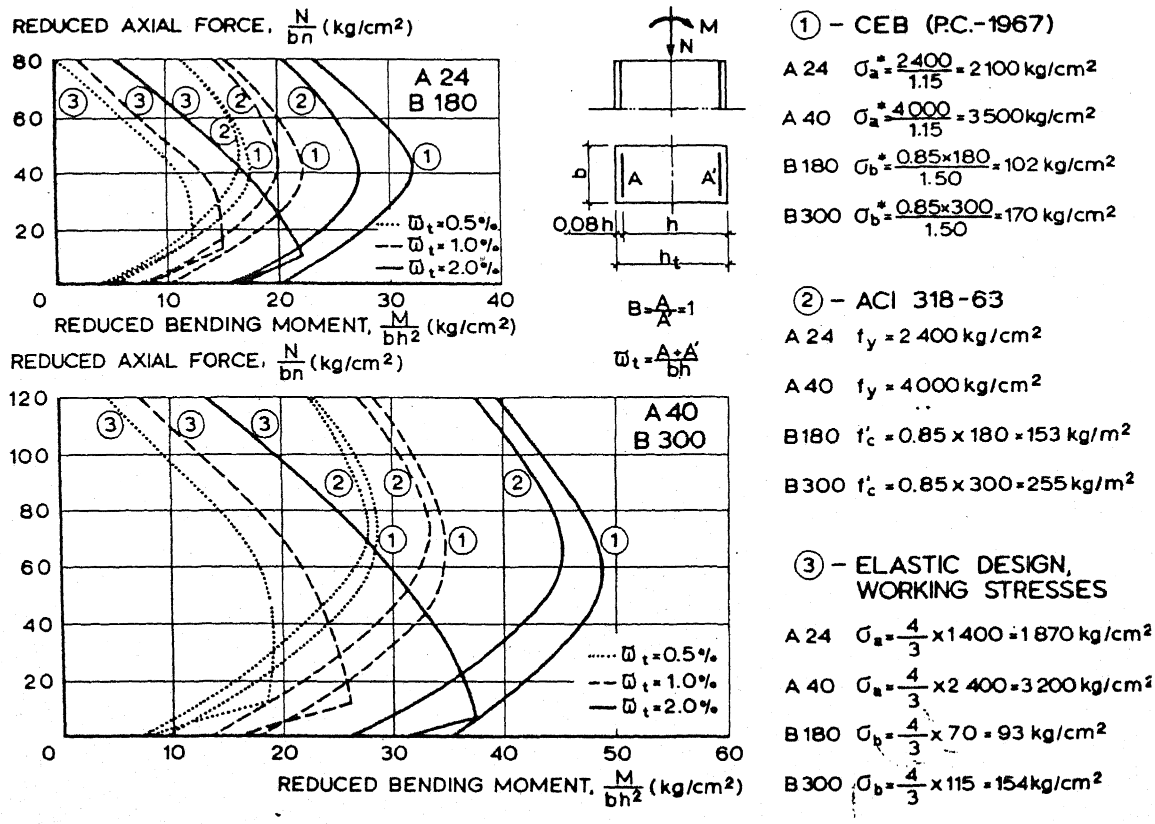

Fig. 2 - Interaction diagrams for reinforced concrete columns

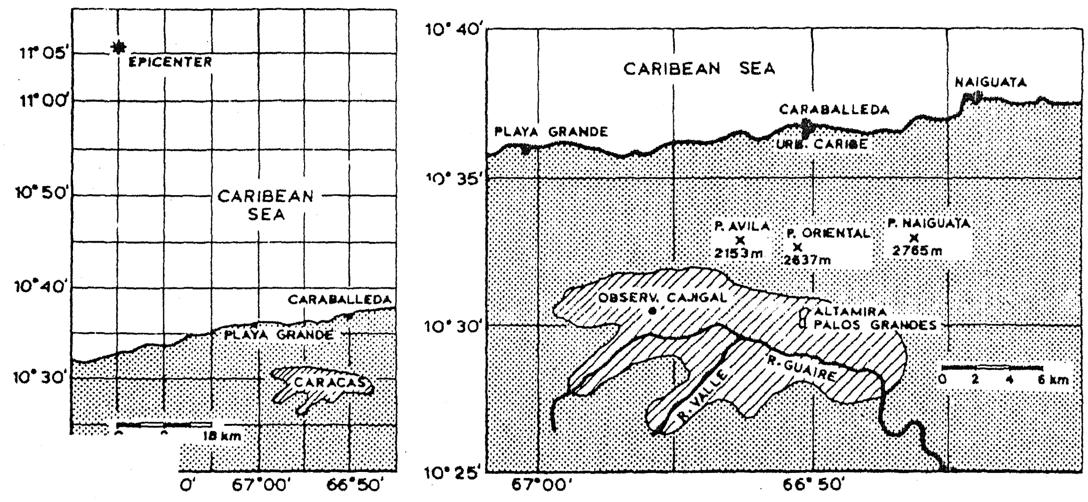

Fig. 3 - Epicenter location. Caracas coastal area

Fig. 4 - Mijagual building

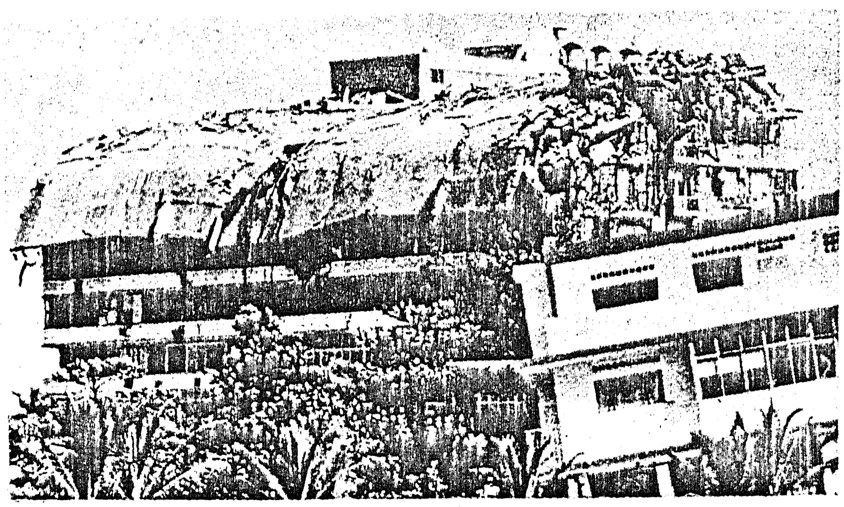

Fig. 5 - Mansion Charaima

Fig. 6 - Palace Corvin

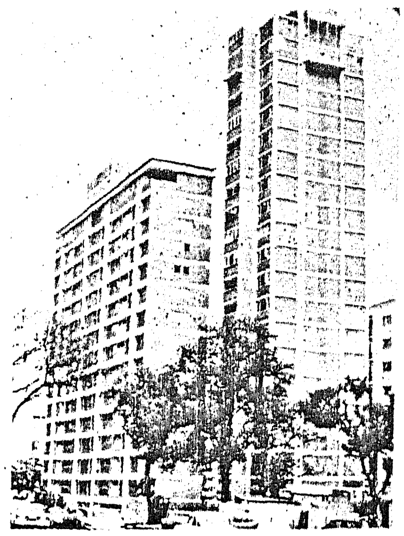

Fig. 7 - Petunia I and II

Fig. 8 - San Bosco

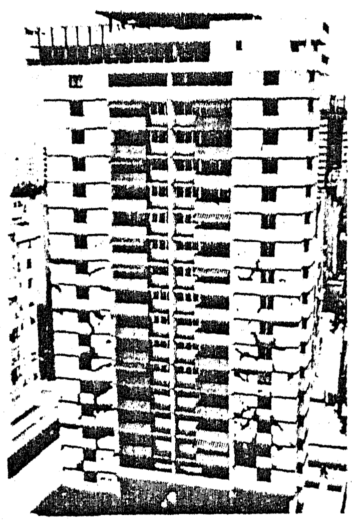

Fig. 9 - Castillete

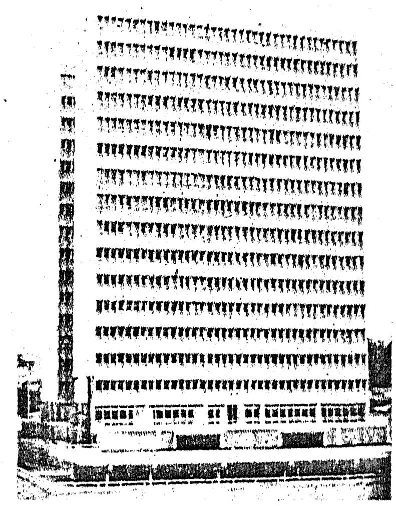

Fig. 10 - Mene Grande

Fig. 11 - Plaza Uno

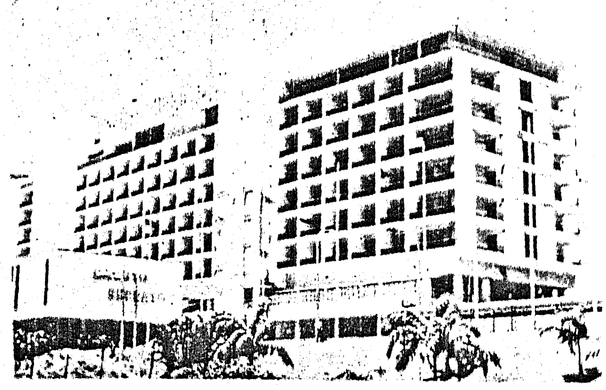

Fig. 12 - Macuto Sheraton Hotel

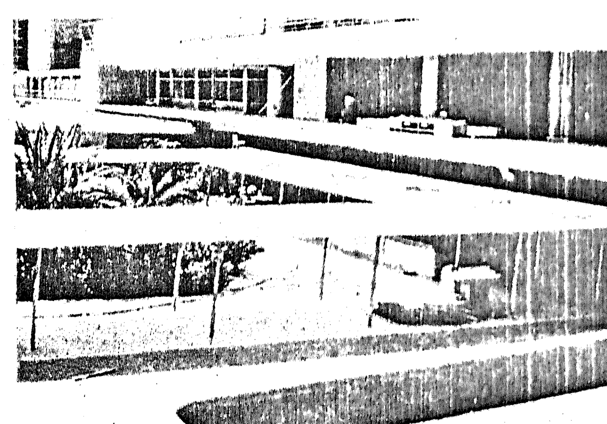

Fig. 13 - Pergola, Macuto Sheraton

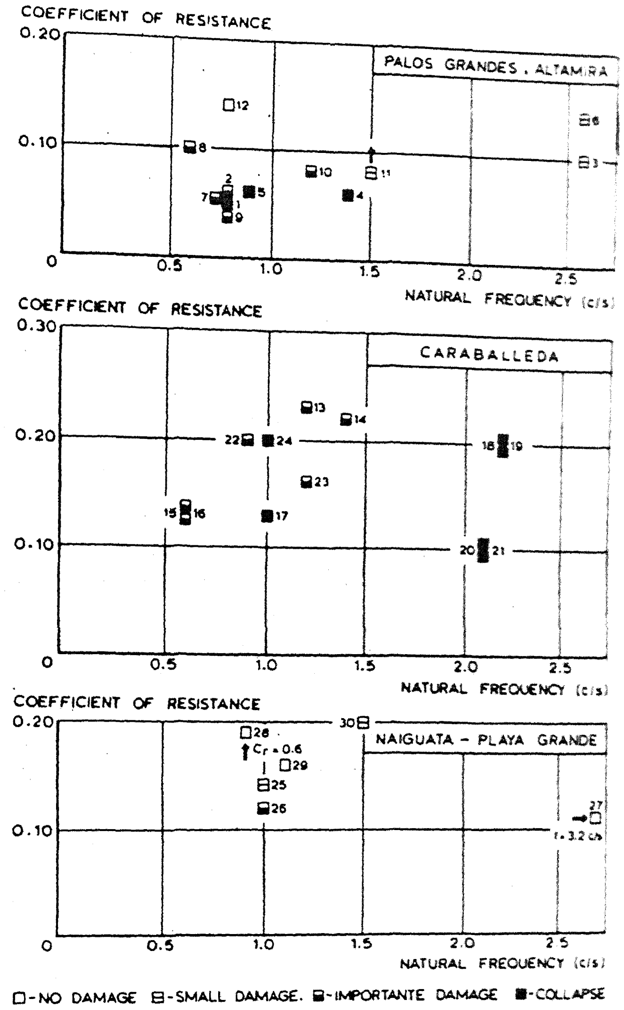

Fig. 14 - Relation between amount of damage coefficient of resistance and computed natural frequency

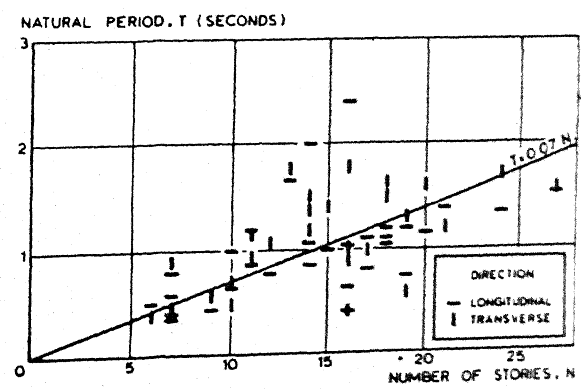

Fig. 15 - Measured periods